CORPS OF ENGINEERS, U. S. ARMY

DRAINAGE SYSTEMS AND ENGINEERING MEASUREMENT DEVICES ST. JOHNS BAYOU FLOODGATE, MISSOURI

TECHNICAL MEMORANDUM NO. 3-425

PREPARED FOR

THE PRESIDENT, MISSISSIPPI RIVER COMMISSION

CORPS OF ENGINEERS

BY

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

JANUARY 1956

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PREFACE

The study reported herein is one of a number of similar studies of the construction and behavior, from a foundation and soil mechanics standpoint, of recently completed structures in the Lower Mississippi Valley Division. These studies are being conducted for the Engineering Division, Mississippi River Commission, by the Waterways Experiment Station.

The drainage and relief well system for the St. Johns Bayou floodgate structure was designed for the Mississippi River Commission by the Waterways Experiment Station. The structure itself was designed by the Mississippi River Commission and built under the supervision of the Memphis District, Corps of Engineers. It was completed 1 April 1953. Installation data and observations made to June 1955 of engineering measurement devices installed at the structure are summarized in this report. It is planned to continue observations of these devices and of the condition of relief wells installed at the structure, and to report any significant data obtained in the form of appendices to this report.

Data on construction procedures and engineering measurement devices were furnished by Messrs. J. W. Black, J. J. Patridge, and J. M. Pollock of the Memphis District, and Messrs. C. I. Mansur, T. B. Goode, and A. L. Mathews of the Waterways Experiment Station. The report was prepared by Mr. R. C. Sloan and reviewed and approved prior to publication by Messrs. W. J. Turnbull, W. G. Shockley, C. I. Mansur, and R. I. Kaufman, Soils Division, Waterways Experiment Station.

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SUMMARY

St. Johns Bayou floodgate is a multiple-barrel drainage culvert in the levee along the Mississippi River at New Madrid, Missouri. It will prevent backwater from flooding the area drained by St. Johns Bayou. The structure consists of six 10- by 10-ft concrete culverts, an intake basin, a control tower, and a stilling basin.

Foundation conditions, relatively high differential heads in both directions, and excavation of approach channels which exposed deep pervious sands, made necessary the installation of drainage blankets and relief wells at each end of the structure to intercept potentially dangerous underseepage. The drainage blankets under the intake and stilling basins consist of a two-layer filter of sand and gravel with perforated collector pipes. Most of the wells consist of 4-1/2-in. ID perforated, flush joint, VC pipe inside of 6-in. ID porous concrete pipe; the wells under the structure proper have 5-in. ID Everdur metal screen. Flap gates are installed on the outlets for the drainage systems to prevent backflow into the systems during reversal of head. Pumping tests on the wells indicated specific yields ranging from about 5 to 40 gpm. Piezometers have been installed to measure the hydraulic gradient beneath the structure and uplift pressures beneath the stilling basin. Differential heads to date have not been sufficiently high to test the structure or drainage systems.

The structure is founded on alluvial sands, and has settled from 0 to 1.0 in. since the levee has been placed over it.

DRAINAGE AND ENGINEERING MEASUREMENT DEVICES, ST. JOHNS BAYOU FLOODGATE, MISSOURI

PART I: INTRODUCTION

Purpose of Study

1. This report is one of a number of similar reports* on studies of the construction and behavior, from a foundation and soil mechanics standpoint, of recently completed flood-control structures in the Lower Mississippi Valley Division. The purposes of these studies are to compare field experience and performance of the structures with design predictions and from such comparisons and observations gain information and experience that will be useful in the design and construction of future projects in the Lower Mississippi Valley Division.

Scope of Report

2. A summary of the design studies made of the drainage and underseepage control systems for the St. Johns Bayou floodgate structure, together with certain observations made of their behavior during construction and subsequently to June 1955, is presented herein. Reference should be made to Waterways Experiment Station report, "St. Johns Bayou Floodgate Structure Drainage System, New Madrid, Missouri," dated July 1947, for details not included in this report.

Description of the Structure

3. The floodgate is located across St. Johns Bayou approximately one mile east of New Madrid, Missouri, and one mile north of the Mississippi River at sta 8+50 of the Birds Point-New Madrid setback levee (see

* A list of similar reports is printed on the inside of the back cover of this volume.

vicinity map on fig. 1). During periods of high water on the Mississippi River, the structure will prevent backwater from flooding the area drained by St. Johns Bayou.

4. The structure consists of a six-barrel (each 10 ft by 10 ft) reinforced concrete box culvert approximately 430 ft long, a control tower and stilling basin on the downstream end, a short intake basin on the upstream end, and approach and outlet channels protected with derrick stone and riprap overlying sand and gravel blankets. A plan and profile are included in fig. 1, and an aerial view of the structure and vicinity is shown in fig. 2. Flow is controlled by six mechanically operated vertical-lift gates at the downstream end of the culverts; see fig. 3. In the intake basin and channel are 33 relief wells connected by 18-in. vitrified clay collector pipe. These wells (Y and Z) are to control seepage and reduce uplift that would tend to lift the slab in the intake basin. Beneath the stilling basin are 12 relief wells (X) to control seepage and reduce uplift under the basin as a result of reverse head. These wells are supplemented by a drainage blanket and 8-in. perforated vitrified clay collector pipes. (See fig. 1.)

5. The natural ground surface is at about elev 285.0* in the vicinity of the structure. The elevation of the bottom of St. Johns Bayou at the structure is about 260; the invert of the box culvert is at elev 258 at the upstream end and at elev 257 at the downstream end. The top of the stilling basin base slab is at elev 256. The net grade of the levee is at elev 312; the gross grade of the levee is at approximate elev 315.

Soil Conditions at Structure

6. In March and August 1947, the Memphis District drilled ten shallow general sample and two undisturbed borings at the site which extended to about elev 250. These borings indicated a relatively uniform soil condition: about 30 ft of relatively impervious top stratum

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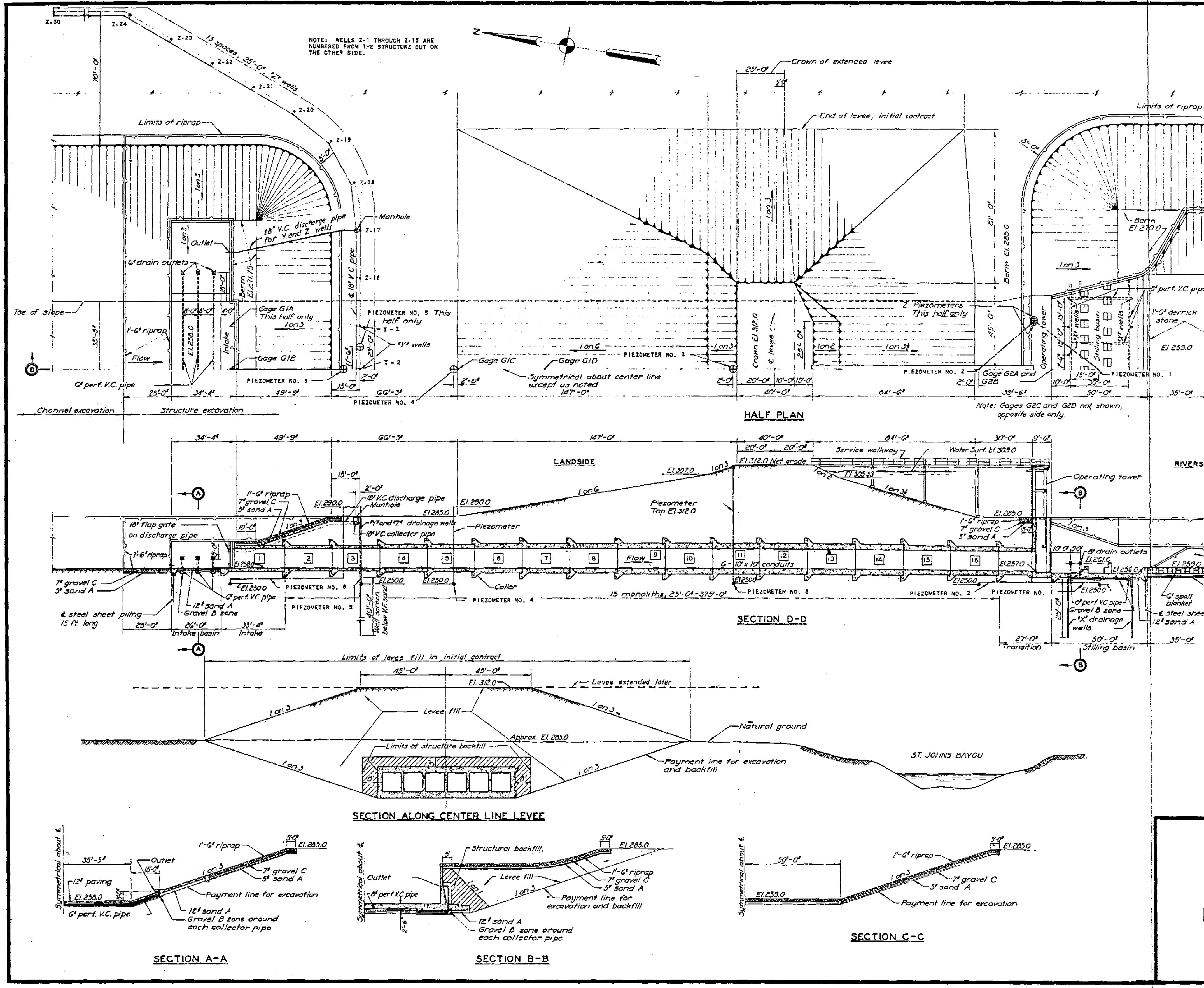
* All elevations are in feet above mean Gulf level.



PLAN, PROFILE AND VICINITY MAP

Fig. 1

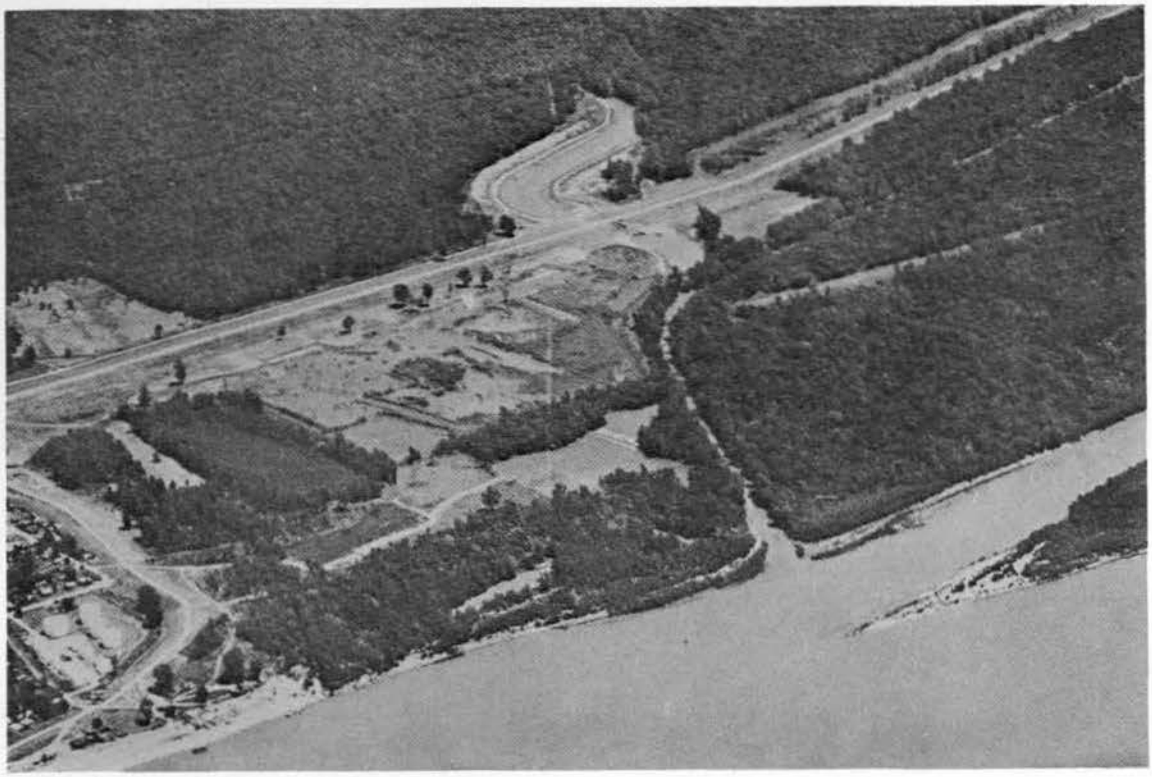


Fig. 2. Aerial view of structure (June 1955)

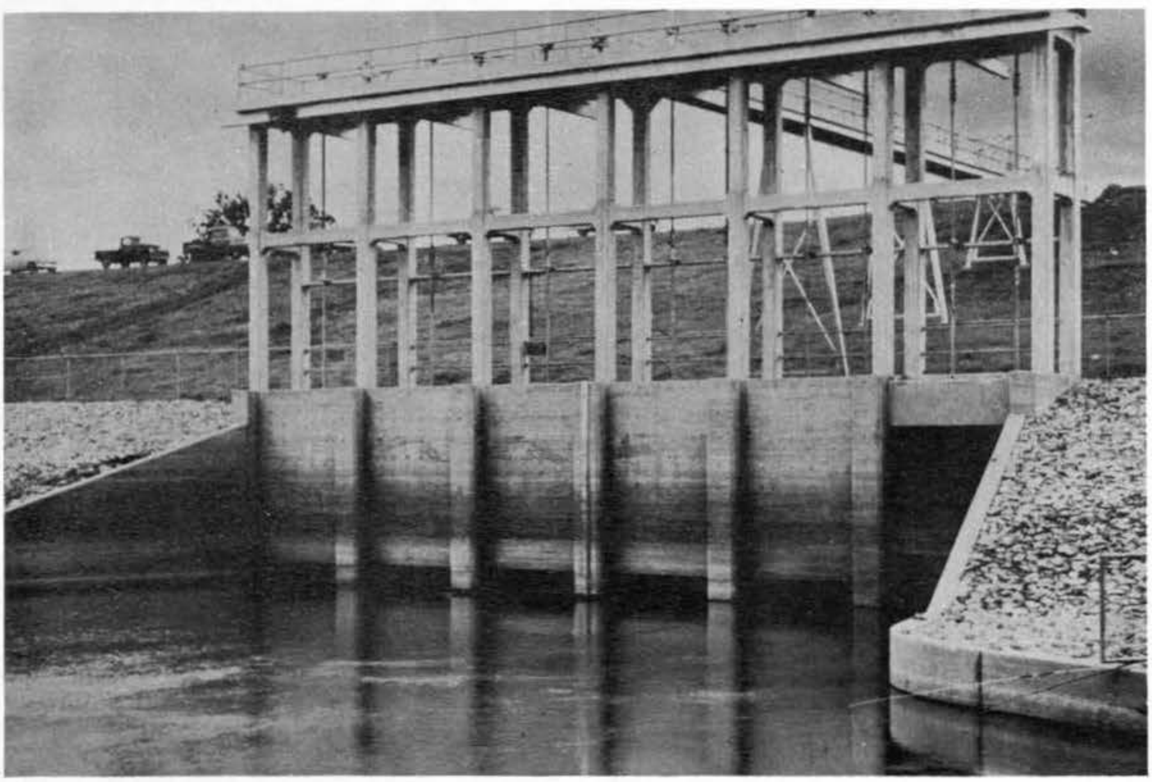


Fig. 3. Downstream end of culverts (July 1955)

composed of clays, silty clays, and silts underlain by pervious substratum sands. Although the thickness of the pervious substratum is not known, other borings in the general vicinity indicate that it exceeds 50 ft, and a thickness of 100 ft was assumed for design purposes. The bottom of the structure, which is at about elev 255, rests on pervious sand generally grading coarser with depth. With such a sand foundation the settlements of the structure would be expected to be small; however, the seepage problem is intensified, as described later.

7. Samples of the foundation sands were not available at the time that the relief wells and drainage systems were designed; therefore, no laboratory tests were performed in connection with the design of these systems.

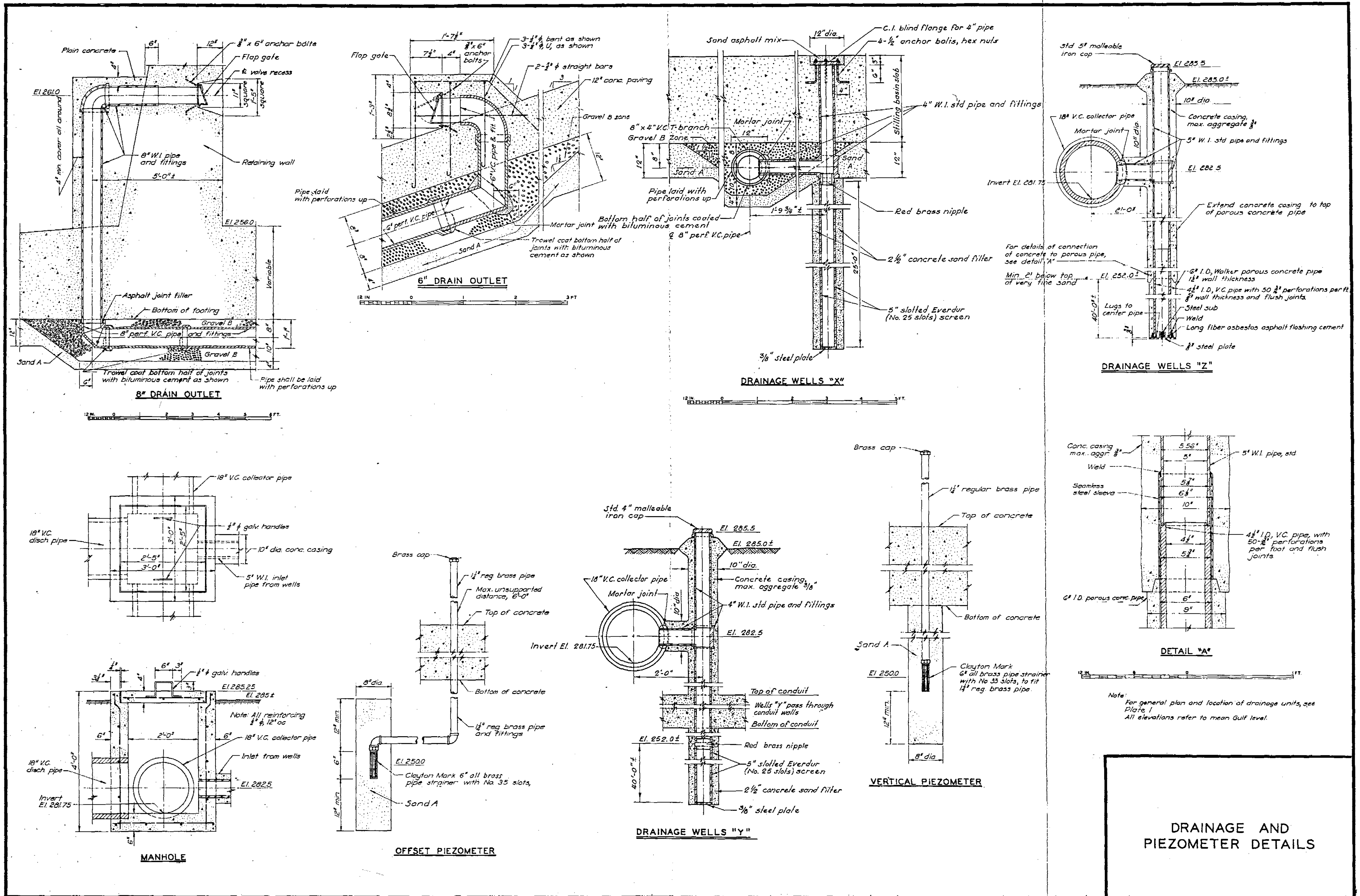
PART II: DRAINAGE AND RELIEF WELL SYSTEMS, AND ENGINEERING MEASUREMENT DEVICES

8. The design maximum net heads on the structure were 24 ft on the riverside and 14 ft on the landside. The slabs for the intake basin and stilling basin were designed assuming no hydrostatic uplift beneath them, and as a result of their limited thickness can only safely withstand excess net heads of about 1.3 and 3.2 ft, respectively, for a factor of safety of 1.1. As the excavations for the inlet and outlet channels completely penetrated the relatively impervious top stratum and exposed the foundation sand, a close seepage entrance is afforded for both design head conditions. Because of the thick top stratum adjacent to the channels, the seepage beneath the structure would tend to concentrate at the ends of the structure, and cause dangerous uplift pressures and piping unless the flow were properly controlled. Therefore, drainage and relief well systems were included in the design to prevent the uplifting of the relatively thin intake and stilling basin slabs and to control seepage beneath the structure.

Intake Basin Relief Well and Drainage Systems

9. Thirty-three relief wells beneath and along the sides of the intake basin were considered necessary to control the seepage in the intake basin area. Assuming a coefficient of permeability of the pervious substratum equal to 500×10^{-4} cm per sec, the depth of the sand aquifer equal to 100 ft, and wells with screens 40 ft long, the required well spacing was computed to be 19 ft for 4-in. ID wells. The wells in the intake basin area are labeled Y and Z and are shown in plan on fig. 1 and in detail on fig. 4. A portion of the intake basin area and the tops of certain Y and Z wells are shown in fig. 5.

10. The screen portion of each Y and Z well is about 40 ft in length. The screens of the Z wells are 4-1/2-in. ID vitrified clay pipe perforated with 50 holes (3/16-in. diam) per linear foot and having a wall thickness of 5/8 in. The perforated clay pipe is encased with a



DRAINAGE AND
PIEZOMETER DETAILS

Fig. 4

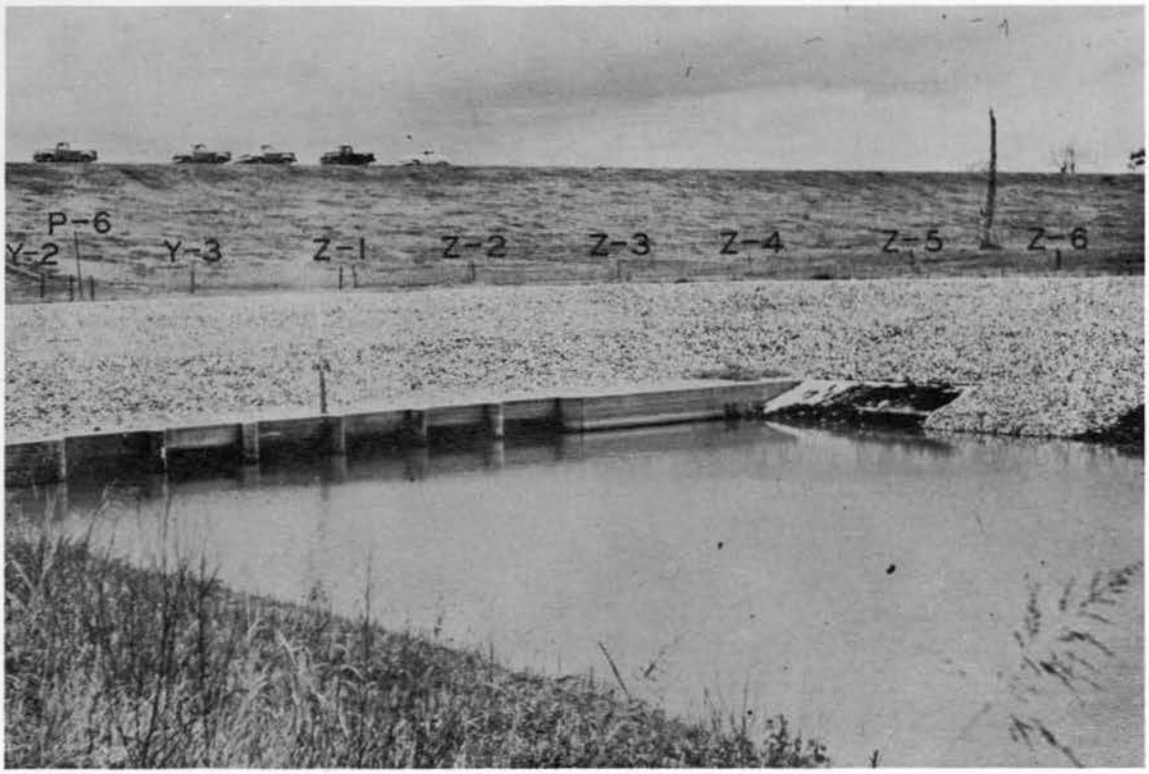


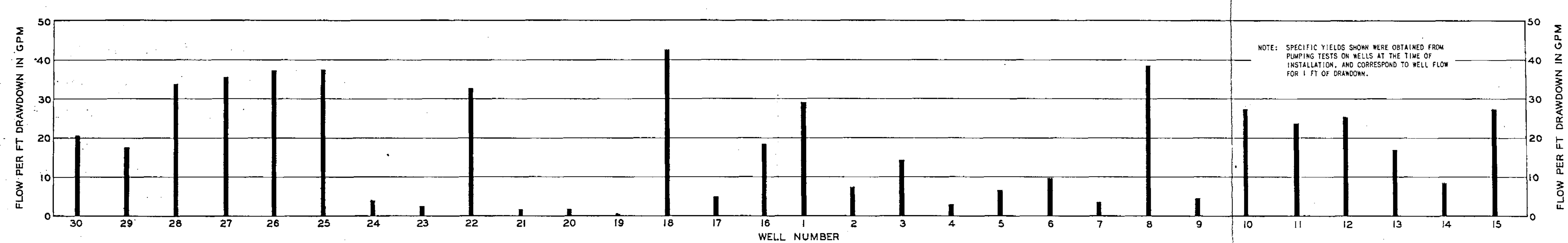
Fig. 5. Intake basin and Y and Z wells (July 1955)

6-in. ID porous concrete pipe that serves as a filter. The joints of the porous concrete pipes are wrapped with heavy glass cloth to prevent sand infiltration at the joints. The riser pipe up to and above the horizontal well outlet pipe consists of standard 5-in. wrought iron pipe. The screens of the Y wells are of 5-in.-diam slotted (No. 25 slot) Everdur metal, and they are surrounded by a 2-1/2-in.-thick filter of concrete sand. The risers are the same as for the Z wells.

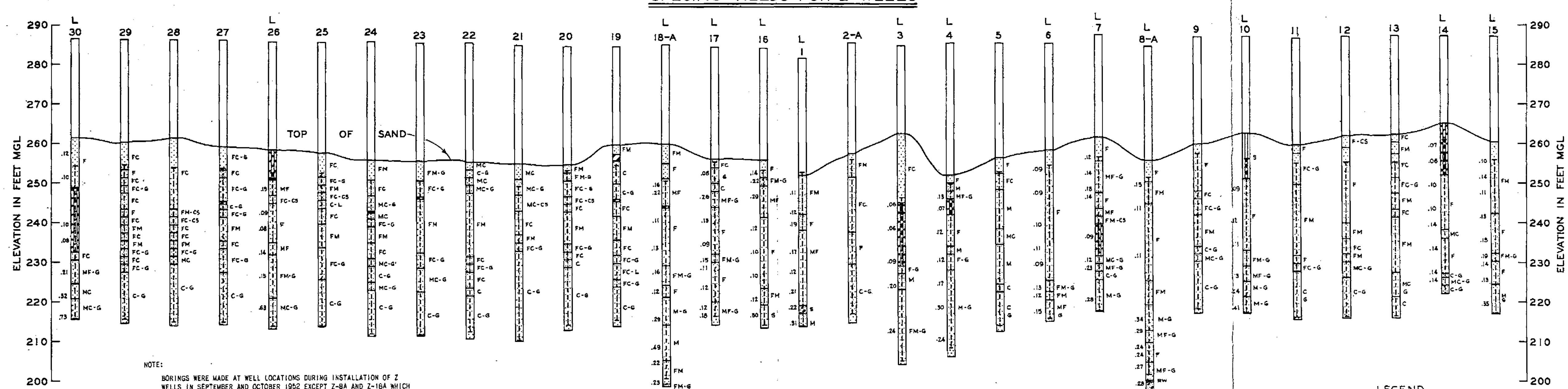
11. Flow from the wells is collected by an 18-in. vitrified clay collector pipe which discharges into manholes, which in turn discharge into another 18-in. vitrified clay pipe that conducts the flow into the intake basin through the wing wall (fig. 1). This outlet pipe is equipped with a Calco flap gate, model No. 100, to prevent silt- and clay-laden water from backflowing into the wells during periods when the wells are not flowing (see figs. 1 and 4).

12. A drainage blanket consisting of 12 in. of sand A drained by horizontal standard strength vitrified clay collector pipes, 6-in. ID with 1/2-in. perforations, and surrounded by gravel B was also installed beneath the intake basin base slab. The collector pipes discharge through the slope paving into the intake basin and have outlets equipped with Pekrul (model No. 16) bronze flap gates to prevent backflow into the sand and gravel blanket. A filter blanket consisting of 7 in. of gravel C underlain by 5 in. of sand A was placed beneath an 18-in.-thick riprap blanket upstream of the intake basin. Limits of the riprap and details of the collector pipes are shown in figs. 1 and 4, respectively; gradation curves of drainage blanket materials are shown in fig. 6. The riprap was required to be of such sizes that not more than 20 per cent of the pieces weighed less than 20 lb and no piece weighed more than 200 lb. The least dimension was required to be not less than one-third the length, and the maximum allowable length was 18 in. The physical requirements were as listed below:

	Minimum	Maximum
Weight per cubic foot	150 lb	
Absorption of water by stones		1.5%
Soundness of stone after immersion		1.25% loss



SPECIFIC YIELDS FOR Z WELLS



NOTE:

BORINGS WERE MADE AT WELL LOCATIONS DURING INSTALLATION OF Z WELLS IN SEPTEMBER AND OCTOBER 1952 EXCEPT Z-8A AND Z-18A WHICH WERE MADE IN MARCH 1954. SAMPLES WERE TAKEN BY BAILER METHOD.

FIGURES TO LEFT OF BORING DENOTE D_{10} SIZE IN MM.

LETTER "L" ABOVE BORINGS INDICATES BORING WAS CLASSIFIED BASED ON LABORATORY DATA; ALL OTHER BORINGS WERE CLASSIFIED IN THE FIELD.

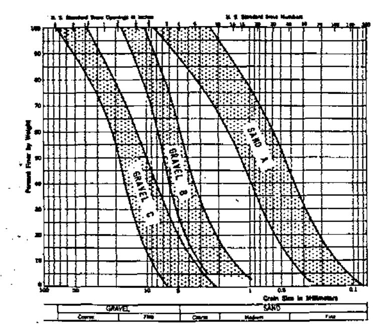
FOR LOCATIONS OF WELLS SEE PLATE 1.

SECTION DEVELOPED ALONG LINE OF Z WELLS

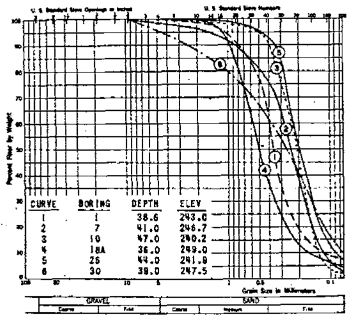
LOGS OF BORINGS

LEGEND

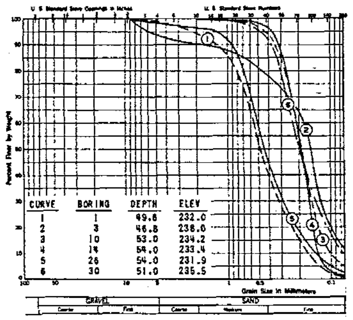
CH	FAT CLAY	
ML	SILT	
SM	SILTY SAND	
SP	SAND, POORLY GRADED	
GP	GRAVEL, POORLY GRADED	
CS	CLAY STRATA	WELL SCREEN
RW	ROTTEN WOOD	
S	SANDY	
G	GRAVELLY	
F	FINE	
M	MEDIUM	
C	COARSE	



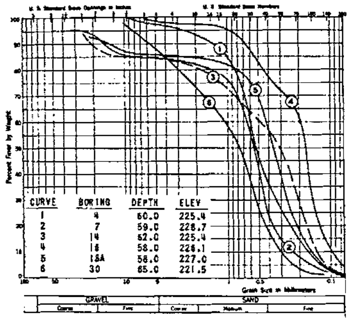
FILTER MATERIALS FOR DRAINAGE BLANKETS



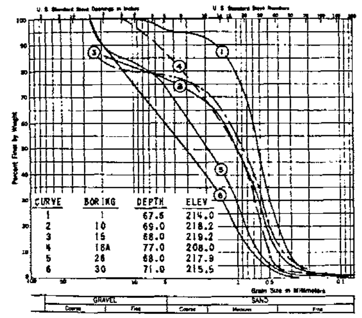
ELEV 250 TO 240



ELEV 240 TO 230



ELEV 230 TO 220



BELOW ELEV 220

REPRESENTATIVE GRAIN SIZE CURVES

LOGS OF BORINGS, SPECIFIC YIELDS, FILTER MATERIALS, AND FOUNDATION SANDS

Fig. 6

13. Steel sheet piling 15 ft long was installed beneath the upstream end of the intake basin slab to insure against undermining of the base slab (see fig. 1).

Outlet Stilling Basin Relief Well and Drainage Systems

14. Twelve relief wells beneath the upstream and downstream ends of the outlet stilling basin slab were considered necessary to control seepage beneath the structure resulting from a possible 14-ft head differential on the landside of the structure. The number and spacing of wells were computed using the same assumed values of coefficient of permeability and depth of sand aquifer as used for the intake basin wells, but with the wells penetrating only 30 per cent of the aquifer. The 12 wells in the stilling basin are labeled "X," and are located on 15-ft centers in two rows 30 ft apart (see fig. 1). The X wells are similar in construction to the Y wells, but have only 25-ft-long screens. Flow from these wells is collected by 8-in., vitrified clay, perforated (1/2-in. holes) pipes which also collect drainage from the blanket beneath the base slab.

15. The drainage blanket consists of 12 in. of sand A with gravel B surrounding the collector pipes as shown in fig. 4. The collector pipes for the drainage wells and system under the stilling basin discharge through outlets in the stilling basin walls; the outlets are equipped with Pekrul (model 16) bronze flap gates. A 3-ft-thick layer of derrick stone, on a 6-in. layer of spalls, 7 in. of gravel C, and 5 in. of sand A, was placed downstream of the stilling basin; in addition, an 18-in.-thick blanket of riprap, on 7 in. of gravel C and 5 in. of sand A, was placed downstream of the area protected by derrick stone (fig. 1). The physical quality requirements of the derrick stone were the same as those for the riprap (paragraph 12). No piece weighed less than 500 lb and the stones were, in general, rectangular in shape with neither the depth nor the width of any stone being less than one-half its length. The riprap specified for the riverside portion of the structure was identical with that specified for the intake or landside

portion. Spalls were required to be fragments of stones equal in quality to the derrick stone and riprap, and no piece weighed more than 25 lb.

Piezometers

16. Six piezometers were installed at the structure by WES personnel. Locations of the piezometers are shown in fig. 1 and details are shown in fig. 4. Piezometer 1 was located beneath the stilling basin slab to check the efficacy of the drainage system beneath the basin. Piezometers 2, 3, and 4 were placed beneath the conduits to obtain the piezometric gradient beneath the structure caused by differential heads. Piezometer 5 was installed midway between wells Y-1 and Y-2 in the intake basin to measure the pressure between the drainage wells during periods of high water. Piezometer 6 was located beneath the landside end of the conduits to determine the efficacy of the drainage system beneath the intake basin base slab.

Settlement Hubs

17. As the structure is founded directly on sand, settlement was expected to occur simultaneously with pouring of concrete and placing of the fill. Hubs were installed in the roof of the east and west conduits to observe the amount of settlement occurring during placement of the fill over the structure. The locations of hubs are given in table 1.

Table 1

Locations of Settlement Hubs, Observed Settlements

Hub No.	Location	Established Elevation (10-30-51)	Settlement in. (11-10-52)
	West Barrel		
TBM 1	13 in. N of center joint between monoliths 1 and 2, 3 ft 10 in. from W wall	267.91	-0.24
TBM 2	12 in. N of center joint between monoliths 4 and 5, 4 ft 8 in. from W wall	267.69	0.00
TBM 3	11 in. N of center joint between monoliths 7 and 8, 5 ft from W wall	267.52	1.08
TBM 4	12 in. N of center joint between monoliths 10 and 11, 4 ft 11 in. from W wall	267.32	0.00
TBM 5	12 in. N of center joint between monoliths 12 and 13, 4 ft 3 in. from W wall	267.24	0.00
TBM 6	24 in. N of center joint between monoliths 14 and 15, 4 ft 11 in. from W wall	267.11	0.00
TBM 7	14 in. N of center joint between monolith 16 and transition unit, 5 ft 3 in. from W wall	267.01	0.00
TBM 8*	Slab of operating tower	------	----
TBM 9	Top of W outlet end wall, 12 ft 9 in. S of expansion joint between wall monoliths 1 and 2	270.01†	
TBM 10	Top of W outlet and wall, 4 in. from S end of wall monolith 3	270.01†	----
	East Barrel		
TBM 1A	15 in. S of where parapet slopes up, 4 ft 7 in. from E wall	267.95	-0.36
TBM 2A	11 in. S of center joint between monoliths 3 and 4, 5 ft from E wall	267.82	0.24
TBM 3A	11 in. S of center joint between monoliths 6 and 7, 4 ft 8 in. from E wall	267.66	0.96
TBM 4A	8 in. S of center joint between monoliths 9 and 10, 5 ft from E wall	267.45	0.60
TBM 5A	14 in. S of center joint between monoliths 11 and 12, 5 ft 1 in. from E wall	267.29	0.36

(Continued)

* Not installed.

† Elevation established August-September 1952.

Table 1 (Continued)

Hub No.	Location	Established Elevation (10-30-51)	Settlement in. (11-10-52)
	East Barrel (Continued)		
TBM 6A	28 in. S of center joint between monoliths 13 and 14, 4 ft 7 in. from E wall	267.14	0.24
TBM 7A	30 in. S of center joint between monoliths 15 and 16, 4 ft 7 in. from E wall	266.99	0.00
TBM 8A*	Slab of operating tower	------	----
TBM 9A	Top of E outlet end wall, 12 ft 9 in. S of expansion joint between wall monoliths 1 and 2	270.04†	----
TBM 10A	Top of E outlet end wall, 11 in. from S end of wall monolith 3		----

* Not installed.

† Elevation established August-September 1952.

PART III: DEWATERING DURING CONSTRUCTION AND RELIEF WELL INSTALLATION

Dewatering

18. Specifications stipulated that construction would begin not later than 1 August 1949 (unless the Mississippi River was above 24 ft above zero on the New Madrid gage, in which case construction would begin when the river fell below that elevation) and that work should be completed within 250 calendar days after commencement, excepting any authorized extensions of time. It was further specified that all work should be performed in the dry. The contractor (McCarthy-Pohl Contractors, Inc., of St. Louis, Mo.) began work on 21 July 1949 and completed the structure on 1 April 1953, having been granted 526 days extension. The extra time was given because adverse weather conditions hindered construction procedures and because the contractor was unable to lower the ground water within the excavated area sufficiently to permit construction in the dry, except during low-water periods.

19. No records, as such, were kept of the dewatering procedure and the contractor was not required to submit a plan to the contracting officer prior to installing and operating the dewatering system. However, informal information available revealed that on 14 November 1949, 501 1-1/2-in. well points were in use and were being pumped by two 12-in. diesel-powered pumps and two 8-in. gasoline-powered pumps, the combined discharge being approximately 9000 gpm. On that date, the water surface in the structure area was reported to be at elev 254 and the river surface was also at elev 254 and rising fairly rapidly. Further pumping indicated that the water in the construction area could not be lowered further with the setup then in use. By 30 November 1949, additional well points and 500 ft of additional header pipe had been installed at elev 255 and the water table had been lowered to grade (approximately elev 252). The Mississippi River was at elev 262 on that date, having been constant at that elevation for at least a week. The contractor then installed two additional pumps in an effort to lower the water table sufficiently

below grade to insure suitable working conditions in the excavation. However, these efforts failed to achieve the desired results and about 1 January 1950 the contractor stopped pumping and the excavation was allowed to flood.

20. On 12 December 1951 a representative of the Waterways Experiment Station visited the site to observe construction procedures and the efficacy of the dewatering system being employed. Piezometer readings obtained on this date are shown in table 2. Piezometer 6 indicated a hydrostatic head of 6.5 ft above the water-surface elevation in the intake basin (which had been flooded to elev 260). The outlets for the three drainage pipes beneath the intake basin slab are at elev 263 and each was discharging approximately 20 gpm on the west wall, which indicated that a hydrostatic head of more than 3 ft existed beneath the 1-ft-thick base slab. The probable reason that the base slab was not raised by the uplift pressure was that the upstream end of the slab was anchored to the shallow steel sheet piling, and support was afforded by the slabs on the adjoining slopes.

21. Piezometer 5 indicated an excess head of about 9 ft above the invert of the conduits. Although this head was not sufficient to raise the conduits, it caused seepage and sand boils along the edge of the structure, particularly at monolith 7 (see fig. 7).

Relief Well Installation and Pumping Tests

22. The relief wells were installed by WES personnel but were connected to the collector pipes by the contractor. The X and Y wells were installed in August and September 1950 by fishtailing down to the fine sands, then casing the hole with 9-3/4-in. ID casing and advancing the hole to the desired elevation by bailing. The screen and riser for the wells then were placed and the casing pulled out.

23. The Y wells were pumped the day following installation by slowly lowering a hose to the bottom of the well and pumping until the discharge ran clear. The hose then was raised in 5-ft stages and the well was pumped at each stage until the discharge was clear; total

Table 2

Piezometer Installation Data and Observations, and Gage Readings

Piez No.	Top of Riser Elev	Center of Tip Elev	Piezometer and Gage Readings							
			1951	1955						
			12 Dec	23 Mar	24 Mar	28 Mar	30 Mar	1 Apr	4 Apr	6 Apr
1	312.21	250.0	261.8	288.2	288.4	290.0	290.2	290.3	289.4	288.2
2	312.32	250.0	-----	288.3	288.7	290.1	290.4	290.5	289.5	288.3
3	314.08	250.0	-----	288.0	288.2	289.1	290.0	290.0	289.3	288.3
4	295.01*	250.0	264.8	-----	-----	-----	-----	-----	-----	-----
5	295.04*	250.0	266.8	-----	-----	-----	-----	-----	-----	-----
6	294.99*	250.0	266.5	-----	-----	-----	-----	-----	-----	-----
LS gage			260.0**	290.4	290.7	291.1	291.2	291.3	291.5	291.0
RS gage			255.0**	293.4	293.9	295.2	295.3	294.9	292.6	290.2

* Elevations raised from 290.0 since the 1955 high water readings.
** Gages in the cofferdam area. River stage at 284.8.

Numerous sand boils upstream of conduits (9 June 1951)

Large boil along east side of conduits at monolith 7 (12 Dec 1951)

Fig. 7. Sand boils that occurred during construction of St. Johns Bayou floodgate

pumping time was about 3 hours per well. (The amount of sand entering the wells during pumping is given in table 3.) The wells were then pumped clean after the tests. As neither well discharge nor drawdown in the well was measured, specific yields cannot be given and estimates of permeability could not be made. The X wells were surged after installation using a disc made from 5/16-in. belting material on a 3/4-in. pipe. This device was surged up and down vigorously at different elevations in the well. The amount of material surged in the well was measured and is shown in table 3. The sand was then pumped out of the wells.

24. The Z wells were installed in September and October 1952 in a manner similar to that for the X and Y wells. While the holes for the Z wells were being advanced, samples of the sands encountered were obtained. The logs of the borings for the Z wells are shown in fig. 6. Values of D_{10} are plotted on the left of the boring logs; representative grain-size curves are also shown. Considerably more difficulties were encountered with the installation of the Z wells than with the X and Y wells. The clay tile pipe used for the well screen was found to be of varying strength, although this could not be detected by superficial inspection. Some sections of pipe were extremely resilient, whereas others shattered under slight impact. A detailed description of all difficulties encountered will not be attempted, but it is worthy of note that 5 of the 30 wells had to be reinstalled because of pipe failures and misalignments caused by the stress induced by the weight of the pipe and the stratified sands closing in irregularly.

25. Installation and pumping test data for the Z wells are shown in table 4 and the specific yields of the wells are plotted in fig. 6. The Z wells were pumped shortly after installation, and the flows and drawdowns in the wells were recorded. It may be noted from table 4 and fig. 6 that the specific yields vary from 0.3 to 42.4 gpm per ft drawdown. On 30 March 1955 a differential head of 4 ft existed on the structure which may have caused the Y and Z wells to flow.

Table 3

Installation, Pumping, and Surging Data for X and Y Wells

Well No.	Bottom Elev mGl	Center of Tee Connection mGl	Top of Riser Pipe mGl	Sand Entering Well during Pumping ft	Sand Entering Well during Surging ft
Y-1	208.5	282.5	285.5	1.0	---
Y-2	208.3	282.5	285.5	1.6	---
Y-3	208.4	282.5	285.5	0.7	---
X-1	224.9	252.9	255.75	---	0.3
X-2	224.8	252.8	255.75	---	0.4
X-3	224.8	252.8	255.75	---	0.3
X-4	224.8	252.8	255.75	---	0.6
X-5	224.9	252.9	255.75	---	0.7
X-6	224.9	252.9	255.75	---	0.6
X-7	224.8	252.8	255.75	---	0.4
X-8	224.8	252.8	255.75	---	0.3
X-9	224.9	252.9	255.75	---	0.5
X-10	224.8	252.8	255.75	---	0.3
X-11	224.8	252.8	255.75	---	0.3
X-12	224.8	252.8	255.75	---	0.4

Table 4

Installation and Pumping Test Data for Z Wells

Well No.	Bottom Elev mGl	Elev Top of Fine Sand, mGl	Well Discharge gpm	Specific Yield gpm/ft
1	213.5	255.7	113.3	29.0
2	216.7	257.4	43.6	7.2
3	204.8	253.4	84.4	14.2
4	208.2	251.6	9.7	2.9
5	213.8	259.4	43.6	6.5
6	215.6	259.0	59.0	9.7
7	217.6	261.0	18.6	3.5
8	201.3	257.1	140.6	38.4
9	218.2	260.2	35.2	4.3
10	217.1	262.7	95.2	27.3
11	216.1	259.8	83.4	23.7
12	216.3	262.1	80.4	25.1
13	216.3	262.4	72.2	17.0
14	222.0	265.4	21.9	8.4
15	216.9	261.5	100.4	27.3
16	213.9	255.6	102.6	18.4
17	216.0	255.8	28.2	4.9
18	200.3	255.4	139.8	42.4
19	215.2	262.6	2.6	0.3
20	213.8	254.4	15.0	1.8
21	210.0	254.9	13.4	1.7
22	212.2	255.4	151.6	32.8
23	211.3	255.5	18.8	2.5
24	211.7	255.9	28.6	3.9
25	213.5	257.8	142.8	37.2
26	213.6	258.4	152.2	37.0
27	214.7	260.0	146.6	35.4
28	214.8	261.2	147.6	33.8
29	215.4	260.3	98.8	17.8
30	215.3	261.5	102.0	20.4

PART IV: ANALYSES OF PIEZOMETRIC AND SETTLEMENT DATA (TO JUNE 1955)

Settlement

26. Elevations of the settlement hubs were obtained shortly after the hubs were installed and also in November 1952. The locations of the hubs and corresponding settlements are given in table 1. It is estimated that the fill was 75 per cent complete in November 1952. The amount of settlement is shown to be negligible except at monolith 7 (see fig. 1 for location) where approximately 1 in. of settlement was observed at hubs 3 and 3A. It is of interest to note that, as previously mentioned, a large sand boil was found in December 1951 on the east side of monolith 7 (see fig. 7(b)); this boil was estimated to be discharging approximately 150 gpm. The comparatively large settlement at monolith 7 might well have been caused by a void under the slab as a result of this sand boil.

Hydrostatic Pressures

27. The highest stage experienced in the Mississippi River since completion of the structure occurred on 28-30 March 1955 and created a net head of 4 ft on the structure with the water surface at about elev 295 on the riverside and 291 on the landside of the levee. Readings were obtained from piezometers 1 through 3 between 23 March and 6 April. These readings are shown, together with riverside and landside gage readings, in table 2. It may be noted that the piezometer readings were lower than the tailwater elevation during both the rise and fall of the river for the 1955 readings. This behavior cannot be explained, unless there was some constant error in the method of obtaining the readings or both upstream and downstream gages are not set at correct elevations. Piezometers 4, 5, and 6 were not read during the 1955 high water; it was intended that extensions be added to the riser pipes before the water reached elev 290, but this was not done and the piezometers were inundated.

Manhole Covers and Flap Gates

28. During a recent visit to the structure by a representative of the Waterways Experiment Station, a considerable quantity of mud was noted at the ends of the 18-in. discharge pipes from the Y and Z wells (see fig. 8). Inspection of the manhole covers revealed that they were not watertight, and silt- and clay-ladened surface water could enter the collector pipes and wells. It could not be determined if the mud observed in the collector pipes had entered through the manholes or was the result of backflow through the flap gates.

29. It was also noted that when the 18-in. Calco gates were opened manually and released so as to swing free, they did not close completely.

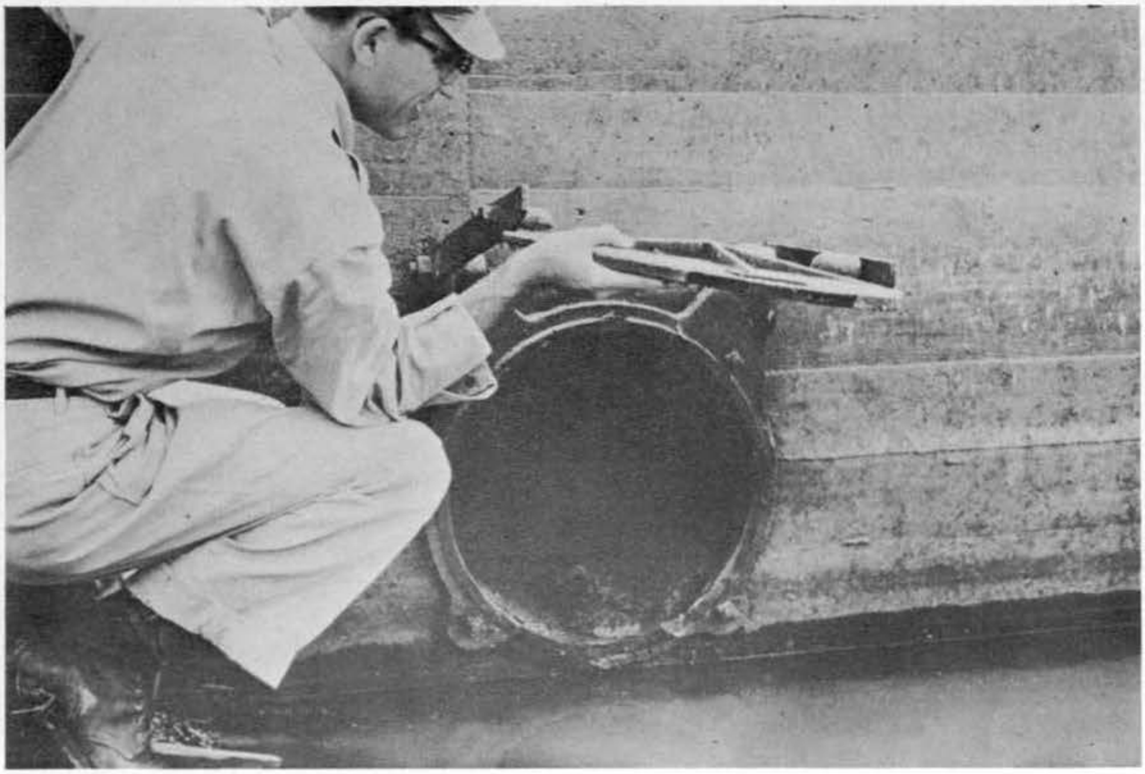


Fig. 8. Mud behind 18-in. flap gate (July 1955)

PART V: RECOMMENDATIONS

30. It is recommended that all wells be sounded to determine the amount of sand that may have entered during the recent high-water period when they were flowing. If an appreciable amount of sand is found, it should be removed. Pumping tests should be performed on the X and Y wells to determine their specific yields for purposes of comparison with future pumping test data. Pumping tests should also be performed on the Z wells, and the specific yields compared to those of the original pumping tests to determine whether the efficiency of the wells has decreased since their installation. It is now believed that this type installation is impractical for future permanent installations except possibly for very shallow relief wells installed in uniform sands containing little or no fine sand.

31. Another survey should be conducted to determine the elevations of the settlement hubs in the east and west conduits and the effect of adding the final fill over the structure (final grade was reached in September 1953).

32. The piezometers should be sounded, and cleaned if necessary. All piezometers should be tested and extensions added to the risers before the next anticipated high-water season. The elevations of the gages and the method of obtaining piezometer readings should be checked as recent readings appear improbable.

33. The present manhole covers should be replaced by metal lids which can be tightened down on rubber gaskets by bolts embedded in the concrete manhole to prevent surface and flood waters from entering the manholes.

34. The 18-in. Calco flap gates should be replaced with bronze-faced and bronze-hinged flap gates, the faces of which should be inclined outward from the vertical to insure proper closing when the wells are not flowing. Rubber gaskets should be installed on the stationary face of the gates to insure a watertight seal when the gates close. Vertical check valves, such as recently developed by the WES and installed on Bayou Cocodrie Drainage Structure, would be more desirable but the cost

involved in their installation on such large pipes may be prohibitive at this structure.

ASSOCIATED REPORTS

Report Designation	Date	Title
MP 3-6	May 1952	Field Observations on Texas and Pacific Main Line Railroad Embankment, Morganza Floodway, Louisiana.
MP 3-17	Sept 1952	Review of Soils Design and Construction of Bayou Rapides Drainage Structure.
MP 3-40	Oct 1952	Review of Soils Design and Construction of Walker Lake Canal Pumping Plant.
MP 3-37	June 1953	Review of Soils Design and Construction of East and West Calumet Floodgates.
MP 3-53	Nov 1953	Review of Soils Design and Field Observations of Bayou Cocodrie Drainage Structure.
TM 3-384	Aug 1954	Review of Soils and Foundation Design and Field Observations, Morganza Floodway Control Structure, Louisiana.
TM 3-409	June 1955	Review of Soils Design and Field Observations, Enid Dam, Mississippi